


To:		ENGINEERING INSTRUCTION <i>New York State Department of Transportation</i>	89-003 <hr/> Supersedes:
Title: SUBJECT METHOD FOR CALCULATING THE LOADS APPLIED TO SPAN WIRE TRAFFIC SIGNAL POLES: NON-TETHERED Subject Code: 7.27-1-680			
Distribution: G Main Office(30) G Local Gov.(31) G Regions(32) G Contractors/AGC(39) G Consultants(34) G _____ ()		Approved: _____ John M. Robb, Asst Deputy Chief Engr. (Structures) 02/27/89 Date	

The following are attached to this E.I.:

- 1) "METHOD FOR CALCULATING THE LOADS APPLIED TO SPAN WIRE TRAFFIC SIGNAL POLES: NON-TETHERED", a design procedure.
- 2) "Traffic Signal Wind Tunnel Test", a magazine article that includes a table with wind loads on traffic signals: and
- 3) a design example using "METHOD FOR CALCULATING THE LOADS APPLIED TO SPAN WIRE TRAFFIC SIGNAL POLES: NON-TETHERED".

The design procedure attached to this E.I. titled "Method for Calculating the Loads Applied to Span Wire Traffic Signal Poles: NON-TETHERED", shall be used to determine the pole load on non-tethered span wire traffic signal poles effective immediately. Prior to the issuance of this E.I., both tethered and non-tethered span wire traffic signal poles were designed according to E.I. 76-43 whose subject is Method for Calculating the Loads Applied to Type A Traffic Poles Carrying Suspended Cables. E.I. 76-43 yielded very conservative results for non-tethered span wire traffic signal poles. The method in E.I. 76-43 will continue to be used for calculating pole loads on tethered span wire traffic signal poles.

It shall also be noted that if, in the foreseeable future, there is a good chance the signal system will be upgraded, the original pole design shall accommodate the expected future loads from signals and/or signs.

This method for determining non-tethered loads shall apply to traffic signals and/or signs suspended on a cable between poles with the ends of the cable attached to the poles at the same elevation. The length of poles need not be equal; however, in such cases, the stiffness of the stiffer pole shall be used to compute the pole loads. To reemphasize, the suspension system shall not include a tether wire strung between the poles when the attached procedure is used.

"METHOD FOR CALCULATING THE LOADS APPLIED TO SPAN WIRE TRAFFIC SIGNAL POLES: NON-TETHERED"

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New York State Department of Transportation
1989

This method shall be used to determine the pole load on non-tethered span wire traffic signal poles, and the pole detection rate range for the calculated pole load.

This method shall apply to traffic signals and/or signs suspended on a cable between poles with the ends of the cable attached to the poles at the same elevation. The length of the poles need not be equal: however, in such cases, the stiffness of the stiffer pole shall be used in compute the pole loads. Again, the suspension system shall not include a tether wire strung between the poles.

- REFERENCES:
- 1) STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS. LUMINARIES AND TRAFFIC SIGNALS 1985 (AASHTO)
 - 2) "TRAFFIC SIGNAL WIND TUNNEL TESTS": THE AMERICAN CITY, BUTTENHEIM PUBLISHING CORPORATION: July 1980 (Included)
 - 3) NEW YORK STATE STANDARD SPECIFICATIONS, CONSTRUCTION AND MATERIALS

DESIGN PROCEDURE

A. CONFIGURATION & LOADS

- Step 1. Determine the span, the location of signals/signs and the magnitude of the signal dead loads. (See Table. Ref. 2. for signal configurations not included on the table, an approximate value can be obtained by interpolation and extrapolation.
- Step 2. Determine location and magnitude of signal/sign wind loads on the cable . (See Table. Ref. 2. For signal configurations not included on the table, an approximate value can be obtained by interpolation and extrapolation.)
- Step 3. Determine location and magnitude of signal/sign ice loads on the cable. (See Ref. 1 Sect. 1, 2, 3.)
- Step 4. Determine the resultant force at each signal/sign location for Group II loading by combining dead load and wind load vectorly. $F=(DL^2 + WL^2)^{1/2}$.
- Step 5. Determine the resultant force at each signal/sign location for Group III loading by combining dead load, ice load and 1/2 wind load vectorly. $F=[(DL+IL)^2 + (1/2 WL)^2]^{1/2}$

B. GROUP I LOADS

- Step 1. Using statics, determine the left and right vertical reactions
- Step 2. Draw a shear diagram to determine the point of zero shear
- Step 3. Set the maximum dead load sag equal to 5% of span. (See Ref. 1, Sect. 1.2.5)
- Step 4. Using statics, determine the horizontal reaction at the attachment point of each pole.
- Step 5. Calculate the lengths of each cable segment due to dead loads.

C. GROUP II LOADS

- Step 1. Using statics, determine the left and right reactions.
- Step 2. Draw a shear diagram to determine the point of zero shear.
- Step 3. Assume a sag for the group loading. (A good first estimate for Group II loading is 120% DO sag.)
- Step 4. Using statics, determine the horizontal reaction at the cable attachment point on each pole for the group loading.
- Step 5. Using cable segment length from Step B-5, calculate the horizontal lengths of each cable segment due to the group loading and the overall length of the span to determine the deflection of each pole. (Maximum pole deflection = $0.6"/l.f.$ See Ref. 3 Section 724-03)
- Step 6. Calculate the minimum deflection rate of each pole due to the added wind load. If the deflection rate is greater than $0.6"/100$ assume a smaller sag and return to Step C-4.

D. GROUP III LOADS

- Step 1. Repeat Steps C 1-6 with Group III loads.
- Step 2. Compare the pole deflection rates for Group I and Group III loadings. If the pole deflection rates are within 15% of each other, proceed to the next step. If the difference is greater than 15%, assume a smaller sag and return to Step 4 for the group loading with the larger pole deflection rate (See example Step 7, Page 9A).

Since pole selection is based on pole deflection rate and the horizontal force, it is important to compare horizontal forces for poles with similar stiffnesses (i.e., poles with nearly equal deflection rates). By keeping the pole deflection rates within 15% of each other, it will reduce the change of having a controlling horizontal force that will not occur because of a higher pole deflection rate from the other group loading.

E. POLE SELECTION

- Step 1. Tabulate Group Loadings, horizontal forces and pole deflection rates.
- Step 2. Calculate the minimum load capacity at Yield Point for the established range of pole deflection rates.
- Step 3. Select a pole that will meet the requirements for each group loading, including:
 - a) Minimum Load Capacity at Yield Point.
 - b) Minimum and Maximum Pole Deflection Rate.
 - c) Maximum Pole Deflection.
 - d) All requirements of Ref. 3 Section 724-03. Traffic Signal Poles.

Traffic signal wind tunnel tests

... assure that you select the correct pole for each installation

What is the wind load caused by a traffic-signal light in a 120 mile per hour wind? Until very recently you probably could not get a very accurate reply. Yet the answer could make a big difference in the type of signal lightpole you buy and install. Now, because of a series of wind tunnel tests, this should become less of a problem for traffic engineers. Moreover, the tests could lead to better designed signal heads.

Previous wind-tunnel tests have established wind loads for most street lighting luminaires. The same did not hold true for traffic-signal lights. This bothered the officials at Hapco, Abingdon, Va., a major manufacturer of aluminum poles used to suspend these lights. So the firm decided to fill the void through the wind-tunnel test technique.

The Aerospace Engineering Department at Virginia Polytechnic Institute, Blacksburg, Va., conducted the tests for Hapco. The largest of three wind tunnels at VPI served as the test site. Hapco Project Engineer R. C. Minor worked with Dr. F. R. DeJarnette, Dr. J. F. Marchman, and W. P. Harrison of VPI in planning, organizing and carrying out the test program. Crouse-Hinds Co., Syracuse, N.Y., supplied the traffic lights. The tests involved several full-size free swinging signals.

The researchers felt it would be impractical to test all possible sizes and combinations of signals. So they selected certain representative samples. The data collected could then be used to make reasonably accurate predictions of the wind

load on many other sizes and combinations not tested.

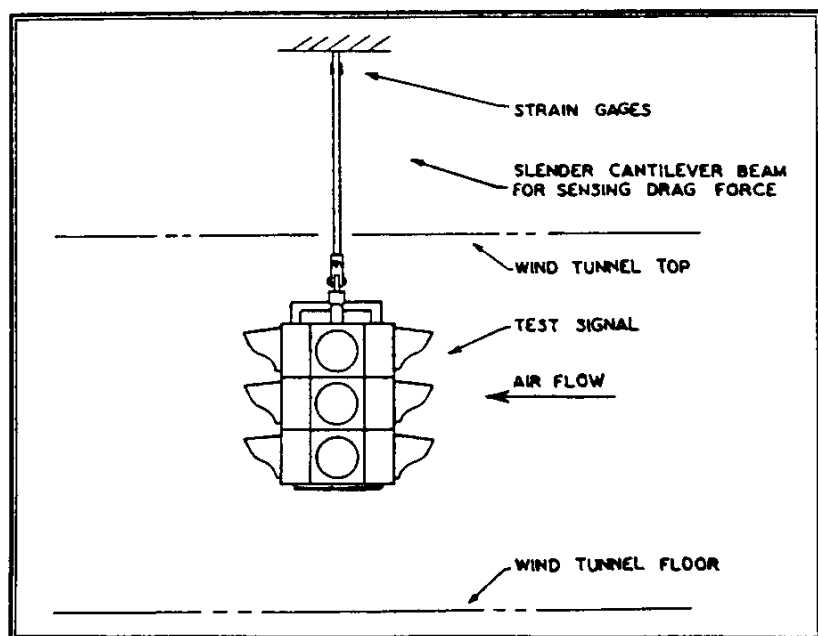
Tests included 10 different three-section adjustable signals. Both 8-inch and 12-inch lens sizes and standard and extended hoods were used. Extra weight added internally to one signal helped to determine its effect on wind load. Also, some signals were rotated and tested in different positions. In all, the men conducted 18 tests, including three of a preliminary nature.

A slender cantilever beam mounted vertically above the tunnel so that it extended down through the roof supported the test signals. (See sketch.) Strain gages mounted on the beam remained

sensitive only to the bending movement applied to it by the wind drag force of the signal. A strip chart recorder, calibrated by applying known horizontal forces, provided a written record of the tests.

Wind velocity varied from 0 to 150 mph. Drag force and velocity readings were taken at specific velocity intervals. A pitotstatic tube measured the dynamic pressure. Windows in the side and top of the tunnel allowed the researchers to observe the behavior of the signals during the tests.

Contrary to expectations, the direction of wind, type of hoods and added internal weight had little effect on the wind force of the sig-



This diagram shows how the wind tests were conducted.

not once the velocity exceeds 75 mph. This means that wind direction would have little if any value in the design of a particular traffic signal.

Each test also involved the plotting of wind force versus wind velocity. At velocities above 80 mph, these curves, for all practical purposes, became straight ones. Therefore, straight-line interpolation and extrapolation could be used to estimate wind loads on signals not tested. (See the accompanying table.)

Most signals tested remained very stable throughout the entire velocity range. At any given speed, these signals assumed a definite angle relative to the wind. They stayed in this position with little motion in any direction. Some signals moved through an angle of nearly 90° or horizontally at velocities of 80 mph or more.

Certain signals with extended full cylinder hoods oscillated in 30 to 80 mph winds. Very unstable behavior occurred when testing the eight-inch, three-section, one-way signal with extended hoods perpendicular to the wind. At about 60 mph, the oscillation became so violent that the test was stopped before the signal destroyed itself against the top and sides of the tunnel. This behavior could produce significant dynamic loads on its supporting structure.

Test use

At present, no generally accepted design standards govern the structured design of signal poles. As a result, each manufacturer has developed his own criteria. The criteria used here, developed by Hapco and based on extensive tests, involves two major factors. They are:

- wind loads on the structure at the maximum wind velocity.
- allowable levels of stresses in the structural members caused by these wind loads.

Before you can establish wind loads, you must first determine the maximum wind velocity to which the structure might be subjected. The United States Weather Bureau's wind map will help gage this factor. The map shows the "isotachs of extreme mile at 30 feet above the ground—for a 50 year mean recurrence interval." The

Wind loads on free swinging traffic signals				Wind load in lbs. @ following velocities		
Lens Dia.	Traffic Signal Size		Weight (lbs.)			
	No. of Sections	Directions		104 mph	117 mph	130 mph
8-8-8	3	1-way	33	39	44	49
8-8-8	3	2-way	71	88	98	105
8-8-8	3	3-way	107	118	140	169
8-8-8	3	4-way	135	147	173	201
12-12-12	3	1-way	49	59*	69*	73*
12-12-12	3	2-way	106	133*	147*	156*
12-12-12	3	3-way	157	178*	210*	250*
12-12-12	3	4-way	204	222	259	298
8-8-8-8	4	1-way	44	53*	59*	65*
8-8-8-8	4	2-way	95	104*	128*	140*
12-12-12-12	4	1-way	65	78*	88*	96*
12-12-12-12	4	2-way	140	168*	189*	208*
12-8-8	3	1-way	40	48*	54*	60*
12-8-8	3	2-way	83	100*	113*	125*
12-8-8-8	4	1-way	50	60*	67*	74*
12-8-8-8	4	2-way	106	127*	143*	157*

* These values were derived from wind tunnel test data

map values represent maximum sustained wind velocities. They should be multiplied by a factor of 1.3 to obtain the maximum gust velocity.

The American Society of Civil Engineers, in a technical paper entitled "Wind Forces on Structures," recommends the use of gust velocity in the design of this type of structure. Once determined, the wind loads caused by wind acting only on the pole shaft and bracket can be calculated from existing theoretical published material and the previous wind tunnel data.

Hapco has established from these findings allowable stress values based on the type of material used, the load and the type of joint involved. For example, higher allowable stresses seem justified for a socket-type joint than one that is a tube welded to a flat plate. Also, stresses caused by deadweight receive very low values compared with allowable stresses caused by high velocity winds.

Once allowable stresses are known, you can calculate the maximum allowable loads which can be applied to any signal pole. Then, you can deduct the wind load caused by wind acting on the pole shaft and bracket arm from the maximum allowable load. This will give you the maximum wind load that can be

applied to a traffic signal mounted on the pole.

Des Moines, Iowa, provided one of the first opportunities to use the wind tunnel data. The city needed a pole with a 25-foot arm to support an eight-inch, three section, four-way signal.

On the wind map, Des Moines lies between the 80 and 85 mph isotach line. The higher isotach has a wind gust velocity of 111 mph. The table does not list wind loads at this rate of speed. However, by straight line interpolation, you can determine the wind load for this particular type of installation as being 161 pounds.

Structural analysis showed that a 25 foot truss arm with upper and lower spars made from ovalized 6- x 0.125-inch tubing of 6063-T6 aluminum alloy would withstand such a wind load. Other design considerations resulted in the selection of a pole shaft of 6063-T6 alloy with a bottom diameter of 10 inches, a top diameter of 8 inches and a wall thickness of 0.250-inch.

This application illustrates the ease and accuracy of selecting the right signal pole using the wind tunnel test data. If you would like more information on these tests, contact Ray C. Minor, Project Engineer, Hapco, P.O. Box 547, Abingdon, Va. 24210. ◀◀

Sheet A of _____ Prep. by _____ Date _____

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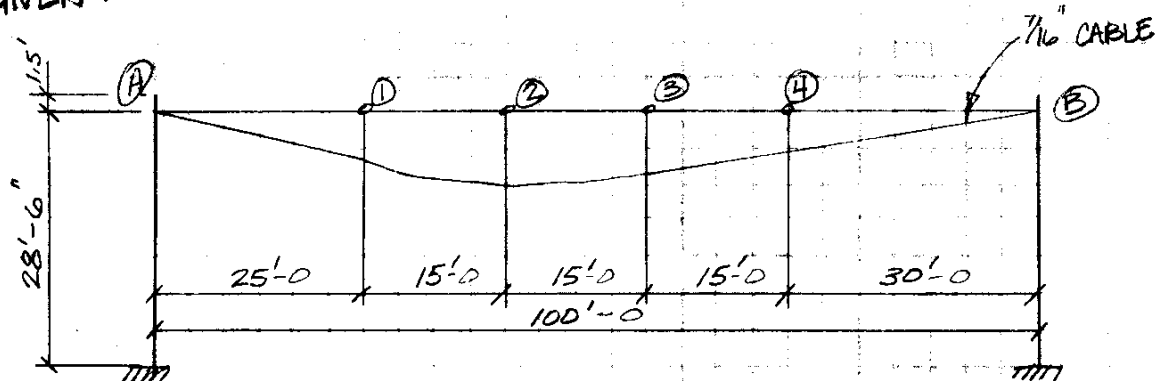
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DESIGN EXAMPLE

GIVEN:



Signals (12" lens diameter)

- ① 3 SECT - 4 WAY
- ② 3 SECT - 3 WAY
- ③ 4 SECT - 1 WAY
- ④ 4 SECT - 2 WAY
- ⑤ 2.5' x 3.0' SIGN @ 45°

WIND SPEED = 80 mph

REFERENCES: STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS
FOR HIGHWAY SIGNS, LUMNAIRES AND TRAFFIC
SIGNALS 1985 (AASHTO)

"TRAFFIC SIGNAL WIND TUNNEL TESTS," THE AMERICAN
CITY, BUTTENHEIM PUBLISHING CORPORATION; JULY, 1970

NEW YORK STATE STANDARD SPECIFICATIONS, CONSTRUCTION
AND MATERIALS

WIND LOAD 104 mph = (1.3×80) ^{fact}

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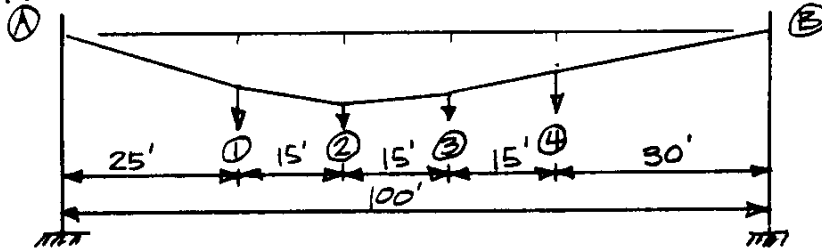
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DESIGN EXAMPLE SOLUTION

A. CONFIGURATION & LOADS

STEP 1.



Loc.	DESCRIPTION	GROUP I D.L.	WL	IL	GROUP II $(DL^2 + WL^2)^{1/2}$	GROUP III $((DL + IL)^2 + (\frac{1}{2}WL)^2)^{1/2}$
①	3 SECT-4 WAY	232	222	156	321	404
②	3 SECT-3 WAY	180	178	117	253	310
③	4 SECT-1 WAY	88	78	57	118	150
④	4 SECT-2 WAY	171	168	110	240	293
④	2.5'x3' SIGN @ 45°	—	175	23	175	91
		<u>671*</u>	<u>821*</u>	<u>463*</u>	<u>1107*</u>	<u>1248*</u>

Note: this table is the results of steps 1-5.

DEAD LOAD = SIGNAL WT. (TABLE, REF. 2) + 8* HANGER + 1*lf (2' of CABLE)
 Note: DEAD LOAD OF SIGN ASSUMED TO BE INSIGNIFICANT FOR THIS DESIGN.

$$\textcircled{1} DL = 204 + 8 + \left(\frac{25 + 15}{2} \right) = 232^*$$

[REPEAT FOR LOCATIONS ②-④]

STEP 2. WIND LOADS (SEE TABLE, REF. 2)

SIGNALS:

$$\textcircled{1} WL = 222^*$$

[REPEAT FOR SIGNAL LOCATIONS ②-④]

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STEP 2. (cont.)

SIGNS:

$$\text{AREA} = 2.5' * 3.0' * 0.707 = 5.30'^2$$

$$\text{PRESSURE} = 0.00256 (1.3V)^2 C_d C_h \text{ (SEE REF. 1, SECT. 1.2.5)}$$

$$V = 80 \text{ mph}$$

$$C_d; \frac{L}{W} = \frac{3.0}{2.5} = 1.2 \therefore C_d = 1.19$$

$$C_h; \text{ht.} = 28' \therefore C_h = 1.0$$

$$P = 0.00256 (1.3(80))^2 1.19 * 1.0$$

$$= 32.9 \rightarrow 33 \frac{1}{2}$$

$$\textcircled{4} \text{ WL}_{\text{sign}} = (33 \frac{1}{2}) (5.30'^2) = 175 \frac{1}{2}$$

STEP 3. ICE LOADS (SEE REF. 1, SECT. 1.2.3)

SIGNALS:

$$IL = (12 \frac{1}{2} \text{ lens}) + (0.6 \frac{1}{2} \text{ cable})$$

$$\textcircled{1} IL = (3 * 4) 12 + 0.6 \left(\frac{25+15}{2} \right) = 156 \frac{1}{2}$$

[REPEAT FOR SIGNAL LOCATION ② - ④]

SIGN:

$$\textcircled{4} IL_{\text{sign}} = (2.5' * 3.0') (3 \frac{1}{2})$$

$$= 22.5 \text{ SAY } 23 \frac{1}{2}$$

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STEP 4. GROUP II LOADS

$$\begin{aligned}
 ① &= (DL^2 + WL^2)^{1/2} \\
 &= (232^2 + 222^2)^{1/2} \\
 &= 321^*
 \end{aligned}$$

[REPEAT FOR LOCATIONS ② - ④]

STEP 5. GROUP III LOADS

$$\begin{aligned}
 ① &= [(DL + IL)^2 + (\frac{1}{2}WL)^2]^{1/2} \\
 &= [(232 + 156)^2 + (\frac{1}{2} * 222)^2]^{1/2} \\
 &= 404^*
 \end{aligned}$$

[REPEAT FOR LOCATIONS ② - ④]

B. GROUP I LOADS

STEP 1. REACTIONS

$$R_A = [(30' * 171^*) + (45 * 88) + (60 * 180) + (75 * 232)] / 100$$

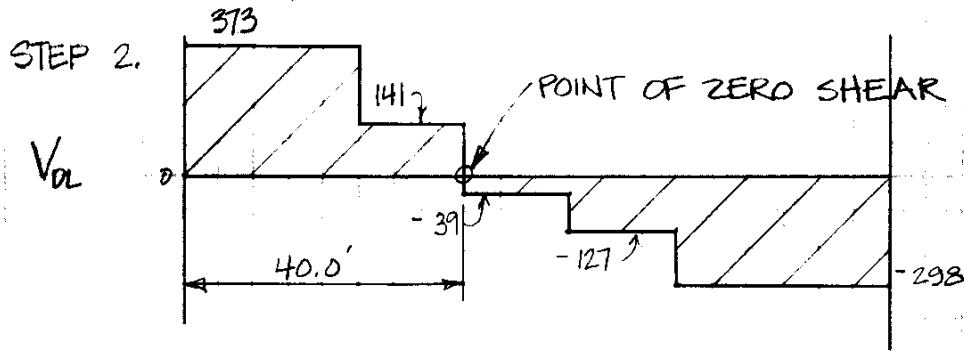
$$R_A = 373^*$$

$$R_B = 671 - R_A$$

$$R_B = 298^*$$

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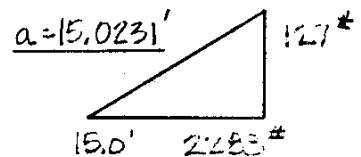
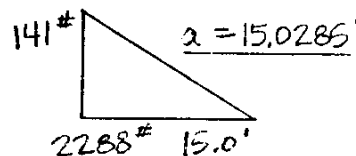
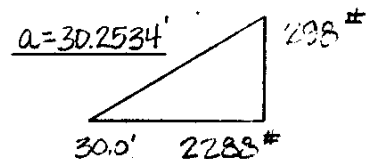
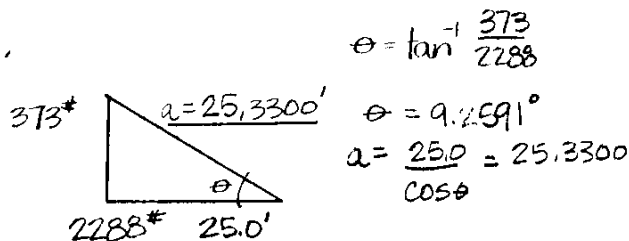
STEP 3. MAXIMUM SAG = 5% SPAN

$$SAG = 0.05 * 100' = 5.0' \text{ (see Ref. 1 section 1.2.5)}$$

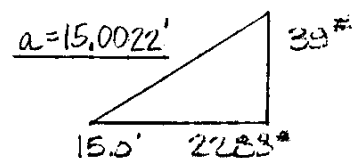
STEP 4.

$$H_A = \frac{[(40' * 373') - (15' + 232')] }{5.0'} = \underline{2288'} = H_A$$

STEP 5.



$$\Sigma a = 100.6372'$$



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SECTION 100-100

C. GROUP II LOADS

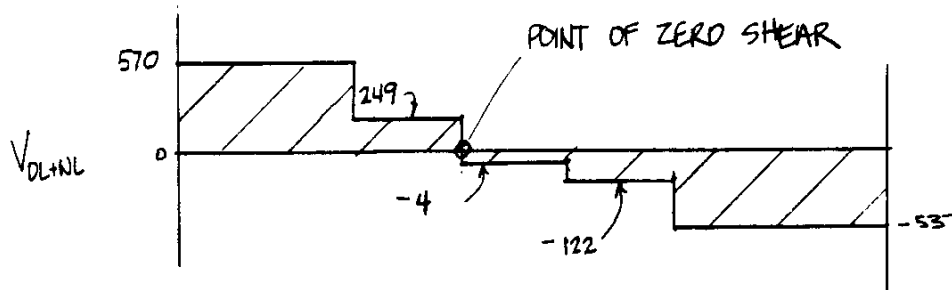
STEP 1.

$$R_A = \frac{[30 \times (240 + 175)] + (45 \times 118) + (60 \times 253) + (75 \times 321)}{100}$$

$$R_A = 570^\#$$

$$R_B = 537^\#$$

STEP 2.



STEP 3. ASSUME SAG (DL+WL) = 120% SAG DL

$$\text{SAG (DL+WL)} = 1.2(5.0) = 6.0'$$

STEP 4.

$$H_A(\text{DL+WL}) = \frac{[(40 \times 570) - (15 \times 321)]}{6.0} = \underline{2998^\#} = H_A$$

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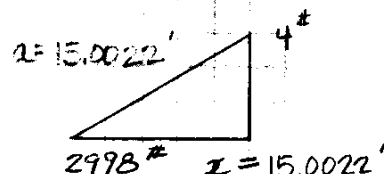
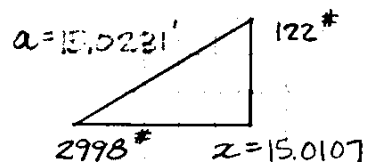
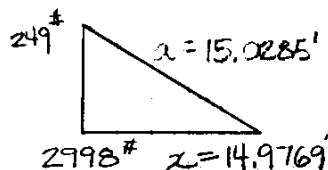
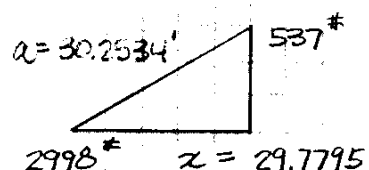
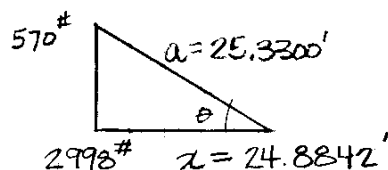
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STEP 5.



$$\theta = \tan^{-1} \frac{570}{2998}$$

$$x = 25.3300' (\cos \theta) = 24.8842'$$

$$\sum x = 99.6535'$$

$$\text{pole deflection}_1 = \frac{100 - \sum x}{2} = 0.173' \Rightarrow 2.079''$$

STEP 6.

$$\text{POLE DEFLECTION RATE} = \frac{2.079' * 100}{\frac{2998 - 2288}{710}} = 0.293 \text{ } \frac{1}{100} \text{ } \text{OK} < 0.60 \frac{1}{100}$$

SELECT A POLE THAT HAS A DEFLECTION RATE GREATER THAN $0.293 \frac{1}{100}$ AND LESS THAN $0.60 \frac{1}{100}$.

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D. GROUP III LOADS

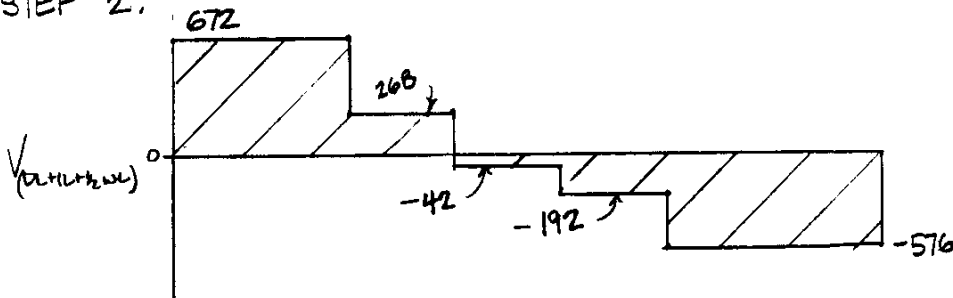
STEP 1.

$$R_A = \left[(30 * (293 + 91)) + (45 * 150) + (60 * 310) + (75 * 404) \right] / 100$$

$$= 672 \#$$

$$R_B = 576 \#$$

STEP 2.

STEP 3. ASSUME SAG $(DL+IL+1/2WL) = 6.3'$

STEP 4.

$$H_A = \left[(40 * 672) - (15 * 404) \right] / 6.3$$

$$H_A = 3305 \#$$

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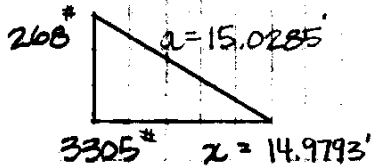
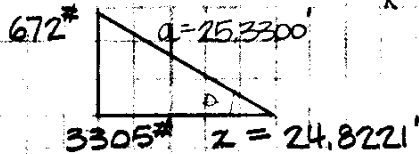
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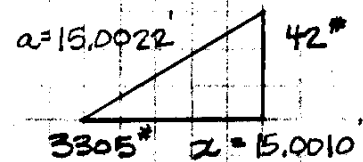
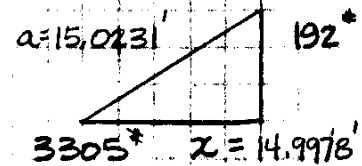
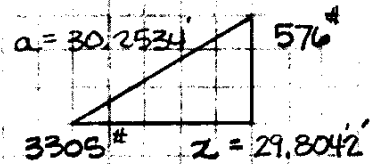
STEP 5.

$$\tan \theta = \frac{672}{3305}$$

$$X = \cos \theta = 25.3300$$



$$\sum x = 99.6044$$



$$\text{pole deflection} = \frac{100 - \sum x}{2} = 0.1978 \rightarrow 2.37''$$

STEP 6.

$$\text{POLE DEFLECTION RATE} = \frac{2.37 * 100}{3305 - 2288} = 0.233\%_{1017}$$

$$\text{STEP 7. } \left(\frac{0.293 - 0.233}{0.233} \right) 100 = 26\% \text{ 715 No Good}$$

∴ ASSUME A SMALLER SAG FOR GROUP II LOADINGS AND
COMPUTE A NEW POLE DEFLECTION RATE

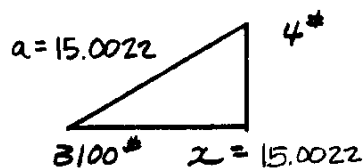
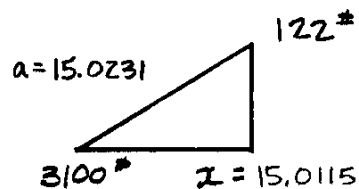
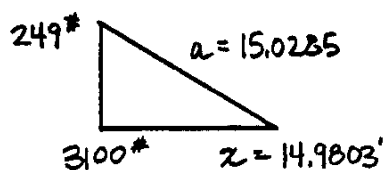
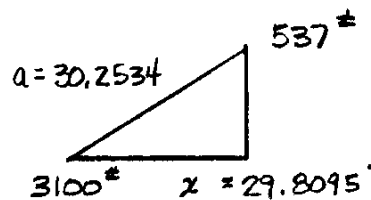
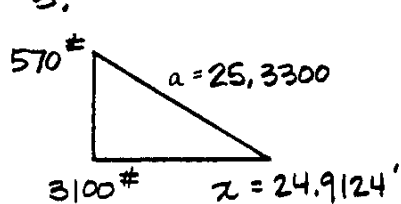
10A of _____ Prep. by _____ Date _____
 _____ Date _____ Design (Ident.) SH No. _____
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 Title _____

STEP 3. ASSUME $SAG = 5.8'$

STEP 4.

$$H_{A(\text{GROUP II})} = [(40 * 570) - (15 * 321)] / 5.8 = 3100\#$$

STEP 5.



$$\Sigma x = 99.716$$

$$\text{pole deflection} = 100 \cdot \frac{\Sigma x}{2} = 0.142' \Rightarrow 1.705''$$

STEP 6.

$$\text{POLE DEFLECTION RATE} = \frac{1.705 * 100}{\frac{3100 - 2288}{812}} = 0.210\%_{100\#}$$

$$\text{STEP 7. } \left(\frac{0.233 - 0.210}{0.210} \right) 100 = 11\% < 15 \text{ ok}$$

Spec. 11A of _____ Prep. by _____ Date _____

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County _____ P.I.N. _____

Job Title _____

CONSTRUCTION JOB STAMP

E. POLE SELECTION

GROUP LOADING	HORIZ. FORCE	POLE DEF. RATE
I	2288 [#]	—
II	3100 [#]	0.210 ^{"/100*}
III	3305 [#]	0.233 ^{"/100*}

 d_p = pole deflection rateMINIMUM LOAD CAPACITY @ YIELD POINT ($0.233\% \leq d_p^* \leq 0.6\% / 100^*$)

$$\text{For Group I} \quad \frac{2288^{\#}}{0.66} = 3467^{\#}$$

$$\text{Group II} \quad \frac{3100^{\#}}{1.4 + 0.66} = 3355^{\#}$$

$$0.924$$

$$\text{Group III} \quad \frac{3305^{\#}}{1.4 + 0.66} = 3577^{\#}$$

$$0.924$$

* THE MINIMUM LOAD CAPACITIES @ YIELD POINT SHOWN ABOVE ARE VALID ONLY FOR POLE DEFLECTION RATES GREATER THAN $0.233\% / 100^*$. IF A POLE WITH A DEFLECTION RATE LESS THAN $0.233\% / 100^*$ IS CONSIDERED, ALL HORIZONTAL FORCES MUST BE RECALCULATED.

POLE SELECTION CRITERIA

a. MINIMUM LOAD CAPACITY @ YIELD POINT = $3,577^{\#}$

b. POLE DEFLECTION RATE: $0.233\% / 100^* \leq d_p \leq 0.60\% / 100^*$

c. POLE DEFLECTION $\leq (0.6' \text{ if pole ht. } (28.5') = 17.1"$

d. MEET ALL REQUIREMENTS OF REF. 3 SECT. 724-03.

